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Engineering Structures 25 (2003) 39–46

ENGINEERING STRUCTURES

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Seismic effectiveness of tuned mass dampers for damage reduction of structures

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Received 16 October 2001; received in revised form 12 June 2002; accepted 26 July 2002

Abstract

The effectiveness of tuned mass dampers (TMD) for control of structures under seismic ground motion is investigated. Since describing the effectiveness of TMD using displacement reduction of the structure is found to be insufficient after yielding of the structure, damage reduction of the structure is proposed instead. Numerical simulations of a 20-storey reinforced concrete building modeled as an equivalent inelastic single-degree-of-freedom (SDOF) system subjected to both harmonic and the 1985 Mexico City (SCT) ground motions are considered. It is demonstrated that although TMD cannot reduce the peak displacement of the controlled structure after yielding, it can significantly reduce damage to the structure. In addition, certain degrees of damage protection and collapse prevention can also be gained from the application of TMD. This is of particular significance in view of the current trend toward performance-based design in which TMD may be installed to achieve the target performance of buildings designed for seismic loads.

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Keywords: Tuned mass dampers; Seismic excitations; Inelastic structures; Damage reduction

1. Introduction

The passive tuned mass damper (TMD) is found to be a simple, effective, inexpensive, and reliable means to suppress undesirable vibrations of structures caused by harmonic or wind excitations [1,2]. Under earthquake excitation, which is rather random, its performance, however, greatly depends on the characteristics of ground motion [3–5]. It was found that TMD consequently becomes effective in reducing the seismic response of structures only when the ground motion exhibits narrow band frequency and long duration.

Villaverde and Koyoama [6] studied the influence of TMD on the response of a ten-storey building subjected to the S60E component of the SCT accelerogram, Mexico City earthquake, 1985. Due to the narrow bandwidth and long duration of the ground motion record, they observed a reduction in the peak roof displacement of the building of about 40% when a TMD with modal mass ratio of 0.042 was installed on its top.

Taking into account the possibility of damage to the building during a high intensity earthquake, Soto-Brito and Ruiz [7] studied the influence of ground motion intensity on the effectiveness of TMD. The response of a 22-storey nonlinear frame with a TMD was considered under moderate and high intensities of SCT accelerograms. Their study showed that the effectiveness of TMD in reducing the peak responses of the structure can be substantially decreased due to nonlinear behavior of the building, which generally occurs under high intensity ground motions.

However, unlike linear systems, describing the effectiveness of TMD using peak response reduction of the nonlinear structures alone was found to be insufficient [8]. Obviously, this measure cannot account for the effects of accumulated damage due to low cycle fatigue. That is of particular importance for the seismic application where the structure may experience a significant number of nonlinear vibration cycles. Therefore, in such circumstances, TMD is expected to effectively reduce not only the peak response of the structure but also the induced-damage of the structure.

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0141-0296/03/$ - see front matter © 2002 Elsevier Science Ltd. All rights reserved.
PII: S0141-0296(02)00115-3

In this study, damage reduction is proposed as an indicator to evaluate seismic effectiveness of TMD for inelastic structures. A 20-storey reinforced concrete building is modeled by an equivalent inelastic single-degree-of-freedom system. The numerical simulations of the building with and without TMD attached on top are performed. Two records of ground motion characterized by a harmonic and a distant earthquake are examined. The inelastic behavior of the structure is introduced by increasing the peak ground acceleration of ground motions. By considering the variation of the energy dissipation in the structure, the TMD's control characteristic and its inability to control high intensity ground motion, as found in the literature, are explained. Since the structure is excited in the inelastic range, the damage is quantified employing the well-accepted damage index [9]. By comparing the induced-damage indices for the structure with and without TMD, the effectiveness of TMD can be evaluated and explained.

2. An example reinforced concrete building

In this study, a 20-storey reinforced concrete office building is considered. The plan and elevations of the building are shown in Fig. 1. The building utilizes a structural system with moment-resisting frames in the longitudinal direction and a coupled moment-resisting frame-shear wall system in the transverse direction. The building is designed only for gravity and wind loads for Thailand with non-seismic detailing. A uniformly distributed live load of 2500 N/m^2 and a maximum wind pressure of approximately 1400 N/m^2 are employed. Compressive strength of concrete of 30 MPa and yield strength of reinforcement of 400 MPa are used. The cross-sections of the shear walls and beams are 0.30 m × 8.00 m and 0.30 m × 0.60 m, respectively, while the dimensions of the columns vary from 0.95 m × 0.95 m at the first floor to 0.45 m × 0.45 m at the roof floor. The program IDARC [10] is used to model the structure. The fundamental period of the structure, determined from analysis in the transverse direction, is found to be 2.13 s. Pushover analysis using an inverted triangular load is also performed. Fig. 2 plots the roof displacement of the structure against the base shear of the structure. It should be noted that response due to designed wind load is well within the elastic range of the curve.

3. An equivalent inelastic SDOF

The 20-storey building outlined in the previous section is modeled by an equivalent inelastic single-degree-of-freedom (SDOF) system. The nonlinear pushover characteristics of the building are ascertained and then its load-deformation characteristic is replaced by the elastic-perfectly plastic behavior of the SDOF system. Although there exist many formulations in the literature for selection of the equivalent inelastic SDOF, the formulation proposed by Fajfar and Fischinger [11] is adopted in this study.

The equations of motion that govern the response of

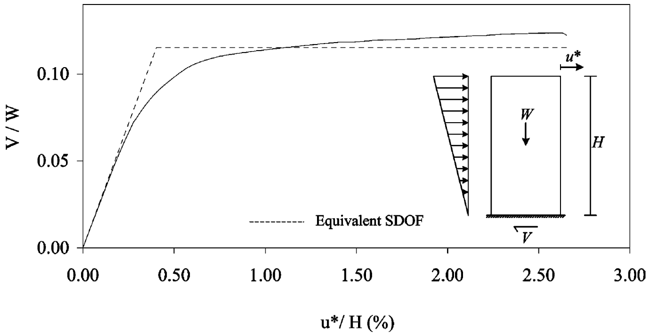

Fig. 2. Pushover curve of structure (----- equivalent SDOF).

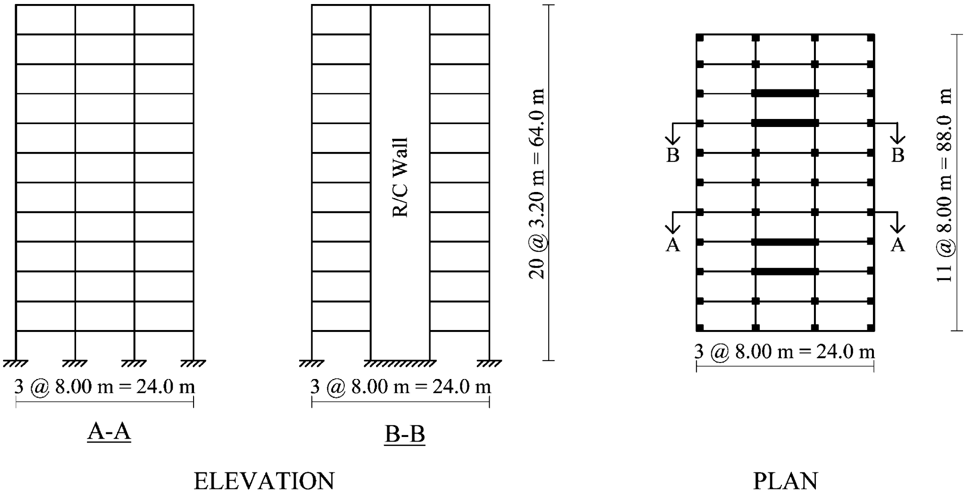

Fig. 1. Elevations and plan of a 20-storey reinforced concrete building.

the structure when subjected to ground excitation can be expressed as

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{Q}(t) = -\mathbf{Mr}\ddot{x}_g(t) \qquad (1)$$

where **M** and **C** are, respectively, the mass and damping matrices of the structure; $\ddot{\mathbf{u}}(t)$and$\dot{\mathbf{u}}(t)$ are, respectively, the acceleration and velocity vectors of the structure relative to the ground; $\mathbf{Q}(t)$ is the storey restoring force vector of the structure; **r** is the influence coefficient vector which represents the displacement vector, $\mathbf{u}(t)$, resulting from a unit support displacement; and $\ddot{x}_g(t)$ is the acceleration of the input ground motion.

The base shear force of the structure is determined from $V(t) = \{\mathbf{1}\}^T\mathbf{Q}(t)$.

Using the transformation of the roof displacement of the structure, $u*(t)$, to the displacement of the SDOF, $x(t)$, by the relationship (Fajfar and Fischnger [11])

$$x(t) = \frac{\Phi^{\mathbf{T}}\mathbf{M}\Phi}{\Phi^{\mathbf{T}}\mathbf{Mr}}u*(t) \qquad (2)$$

in which Φ is the assumed shape vector of the structure normalized with respect to $u*(t)$, the above equation of motion of the structure, Eq. (1), can be rewritten for the equivalent SDOF as

$$m\ddot{x}(t) + c\dot{x}(t) + q(t) = -m\ddot{x}_g(t) \qquad (3)$$

where $m = \Phi^{\mathbf{T}}\mathbf{Mr}$ is an equivalent mass; $c = \Phi^{\mathbf{T}}\mathbf{C}\Phi\cdot\frac{\Phi^{\mathbf{T}}\mathbf{Mr}}{\Phi^{\mathbf{T}}\mathbf{M}\Phi}$ is an equivalent damping; $q(t) = \Phi^{\mathbf{T}}\mathbf{Q}(t)$ is an equivalent restoring force and $\ddot{x}(t)$ and $\dot{x}(t)$ are the acceleration and the velocity of the equivalent SDOF.

Based on the load-displacement relationship of the structure obtained from pushover analysis with an inverted-triangle load distribution as shown in Fig. 2, the corresponding properties of the equivalent elastic-perfectly plastic SDOF previously mentioned can be computed as listed in Table 1 and plotted as the dotted-line in the same figure.

4. Earthquake ground motions

Two different ground motions, characterized by harmonic acceleration and distant earthquake acceleration, are employed as the input ground motions. Both are scaled to have various peak ground accelerations (PGA) in order to study the influence of the degree of inelasticity in the structure on the effectiveness of TMD. Harmonic ground motion is generated for 50 s from a constant amplitude sine wave with the same period as that of the structure (the frequency ratio = 1.0), while the acceleration record at the SCT station of the 1985 Mexico City earthquake, as shown in Fig. 3a, is used to represent distant earthquake ground motion. The dominant period of this acceleration record is very close to that of the structure as indicated by the acceleration spectra in Fig. 3b. These intentionally lead to extreme conditions where the structure vibrates in resonance with the excitations and consequently TMD is expected to be effective for suppression of both displacement and energy of the structure.

5. Damage quantification

For earthquake excitation, it is generally necessary to permit some degree of induced-damage in the structure,

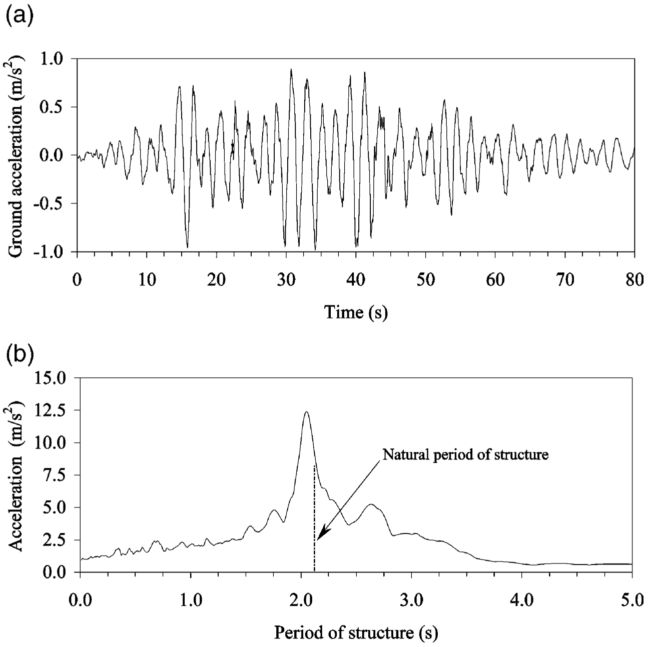

Fig. 3. The 1985 Mexico City earthquake ground motion recorded at SCT station, (a) the acceleration record and (b) the acceleration spectra.

Table 1
Properties of the structure, the equivalent inelastic SDOF and TMD

Structure	System, Eq. (1)	W=378.0 MN, V_y=43.5 MN, u^*_y=0.258 m, u^*_u=1.68 m, ductility=6.5
	SDOF, Eq. (3)	m=1.79 × 10^4 tons, ω=2.95 rad/s, ξ=0.02, q_y=27.4 MN, x_y=0.175 m, x_u=1.14 m $\frac{\Phi^{\mathbf{T}}\mathbf{M}\Phi}{\Phi^{\mathbf{T}}\mathbf{Mr}}$=0.678
Tuned mass damper (TMD)		$m_T = 5.37 \times 10^2$ tons, $k_T = 4.41 \times 10^3$ kN/m, $\omega_T = 2.87$ rad/s, $\xi_T = 0.105$

otherwise the design would be too costly. However, the damage should be kept below a certain amount in order to avoid collapse.

To investigate the effectiveness of TMD in reducing damage to the structure induced by ground excitation, the damage index as suggested by Park et al. [9] is adopted for the system Eq. (1), which can be globally described by

$$DI = \frac{u_{\mathrm{m}}^{*}}{u_{\mathrm{u}}^{*}} + \beta \frac{\int dE}{u_{\mathrm{u}}^{*} V_{\mathrm{y}}} \tag{4}$$

where u_{m}^{*} and u_{u}^{*} are the maximum and the ultimate displacements of the structure at roof, respectively; $\int dE$ is the cumulative hysteresis energy of the structure; V_{y} is the base shear at yielding of the structure and β is the strength deterioration parameter (≈0.27 by Ciampoli, et al. [12]).

This damage equation accounts for damage due to maximum inelastic displacement, as well as damage due to cumulative low-cycle fatigue. The value of DI can vary from 0 to 1.0, which corresponds respectively to the damage level of the structure from no damage to collapse [13].

6. Structure with tuned mass damper (TMD)

A TMD is installed on top of the structure to reduce the displacement and the damage induced by ground excitations. For computational convenience, the equivalent inelastic SDOF obtained in Section 3 is employed to represent the 20-storey R/C building as outlined in Section 2. Thus the equations of motion describing the structure-TMD interaction can be simply expressed by

$$m\ddot{x}(t) + c\dot{x}(t) + q(t) = -m\ddot{x}_{\mathrm{g}}(t) + c_{\mathrm{T}}\dot{z}(t) + k_{\mathrm{T}}z(t) \tag{5}$$

and

$$m_{\mathrm{T}}\ddot{z}(t) + c_{\mathrm{T}}\dot{z}(t) + k_{\mathrm{T}}z(t) = -m_{\mathrm{T}}\left[\ddot{x}_{\mathrm{g}}(t) + \frac{\mathbf{\Phi^{T}Mr}}{\mathbf{\Phi^{T}M\Phi}}\ddot{x}(t)\right] \tag{6}$$

where m_{T},c_{T} and k_{T} are the mass, damping, and stiffness of the TMD, respectively; and $\ddot{z}(t)$, $\dot{z}(t)$, and $z(t)$ are the relative acceleration, velocity and displacement of the TMD.

In the following numerical investigation, a mass ratio of 1.40%, corresponding to an effective mass ratio of 0.03 between the TMD mass and the structure's first-mode generalized mass, is considered. Based on linear behavior of the controlled structure, the stiffness and damping coefficients of TMD are optimized for harmonic base excitation [14]. This yields the optimal values of the TMD parameters as listed in Table 1 along with the properties of the example structure.

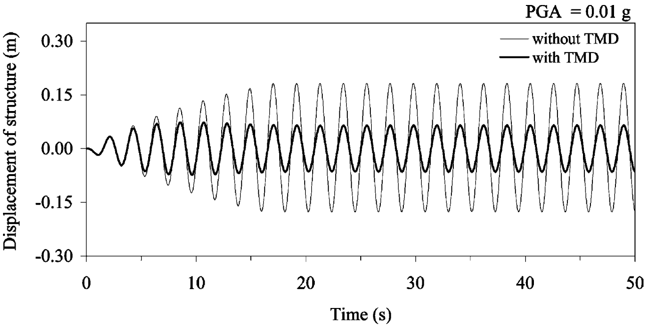

Fig. 4. Displacement histories of structure with and without TMD under harmonic ground motion with PGA of 0.01 g and frequency ratio = 1.0 (case of elastic responses).

7. Effectiveness of TMD under a harmonic ground motion

The effectiveness of TMD is evaluated under harmonic ground motion. The inability to control high intensity ground motion is demonstrated and explained. The use of damage reduction of the structure as the effectiveness measure of TMD is introduced instead of using displacement reduction alone.

7.1. Displacement reduction

Figs. 4 and 5 show the typical displacement histories of the structure with and without TMD under harmonic ground motion with PGAs of 0.01 and 0.05 g, respectively. The motion frequency is intentionally set to be the natural frequency of the structure. In these figures, the former represents the case where the structure is vibrated within the elastic range and significant displacement reduction of the structure can be gained from the application of TMD, whereas the latter represents the case

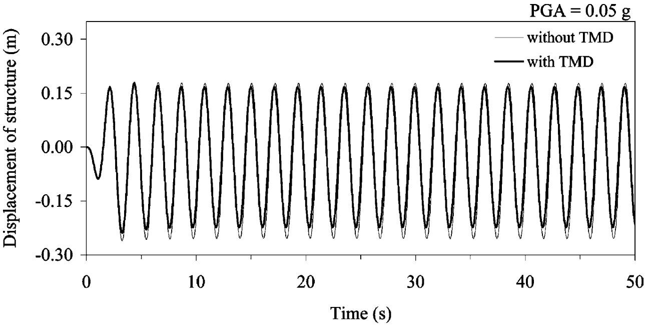

Fig. 5. Displacement histories of structure with and without TMD under harmonic ground motion with PGA of 0.05g and frequency ratio = 1.0 (case of inelastic responses).

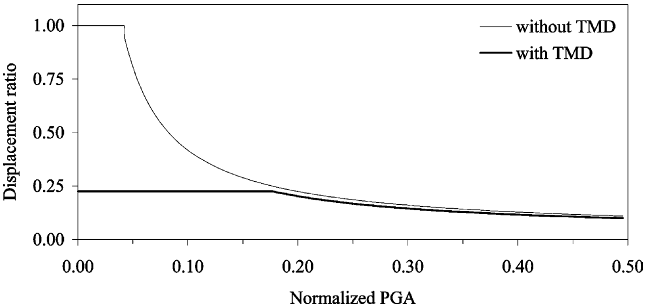

Fig. 6. Displacement ratio of structure against normalized PGA (frequency ratio = 1.0).

where the structure is vibrated well within the inelastic range and the displacement reduction of the structure by application of TMD is substantially decreased. It is noted that, in Fig. 5, the damage indices of the structure with and without TMD become, respectively, 0.55 and 1.0 at the end of the excitation. To show the influence of degree of inelasticity in the structure on the performance of TMD, Fig. 6 compares the displacement ratio of the structure with and without TMD for various PGAs of ground motion. In the figure, the displacement ratio of the structure is calculated from the ratio of the steady-state displacement of the inelastic structure with or without TMD to that of the corresponding elastic structure without TMD, while the input PGA is represented by the normalized PGA, $|\ddot{x}_{\mathrm{g}}|/(q_{\mathrm{y}}/m)$. This quantity reflects both the amplitude of ground shaking and the capacity of the structural system. To ascertain the effectiveness of TMD in terms of displacement reduction of the structure, Fig. 7 plots the steady-state displacement reduction of the structure due to the application of TMD as a function of normalized PGA. In this paper, the displacement reduction is calculated from the ratio of $(|u^{*}|_{\mathrm{w/o\ TMD}} - |u^{*}|_{\mathrm{w\ TMD}})/|u^{*}|_{\mathrm{w/o\ TMD}}$. It is obvious from Figs. 6 and 7 that TMD is very effective in reducing the displacement of the structure by as much as 77% when the normalized PGA is less than 0.042. This is the case where the structure is vibrated within its elastic range. However, beyond this range, the TMD's effectiveness gradually decreases as the PGA increases and becomes as small as 10% when the normalized PGA is greater than 0.176. These findings coincide with those found in previous studies [4,7] in which the effectiveness of TMD is reported to be substantially decreased when the structure's motion is characterized by significant inelastic vibration. This control deficiency can be demonstrated through inspection of the energy dissipation of TMD as shown in Fig. 8. In the figure, the energy dissipation in the structure due to inherent damping, yielding and TMD are normalized by the corresponding kinetic energy of the structure and are plotted against the normalized PGA. The figure shows that TMD loses its effectiveness slightly in dissipating the energy from the structure once yielding occurs in the structure. This is because yielding raises the apparent damping of the structure and consequently disturbs the tuning condition of TMD. Although the dissipation reduction of TMD seems significant in this considered range of PGA, the large amount of dissipation, due to yielding, causes the contribution of TMD to become a comparatively small portion. As a result, a displacement reduction of the structure of only 10% is obtained from the application of TMD.

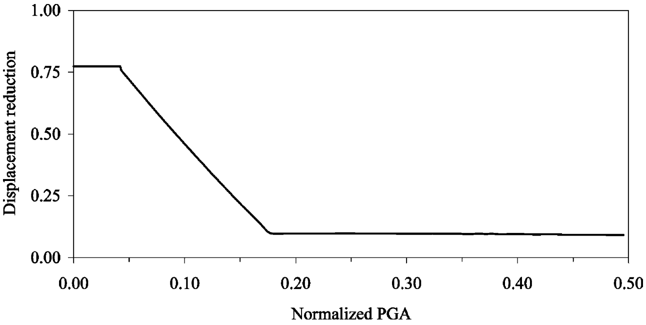

Fig. 7. Displacement reduction of structure with TMD against normalized PGA (frequency ratio = 1.0).

7.2. Damage reduction

It is clear from Figs. 6–8 that TMD can completely protect the building from yielding up to a normalized PGA of 0.156, although the displacement reduction in this range indicates reduction of TMD's effectiveness. This is of particular importance for seismic applications in which the objective of TMD installation is to suppress the damage to the structure rather than its displacement.

Therefore, the following introduces the damage reduction of the structure to describe the effectiveness of TMD instead of the displacement reduction. Employing the damage model previously outlined, Fig. 9 compares the damage index of the structure with and without TMD under harmonic ground motion, while Fig. 10 shows the damage reduction gained from TMD. In the figure, the damage reduction is calculated from the difference between damage to the structure with TMD and without TMD divided by that without TMD, $(DI_{w/o} - DI_{TMD})/DI_{w/o}$. It is pointed out that, for nor-

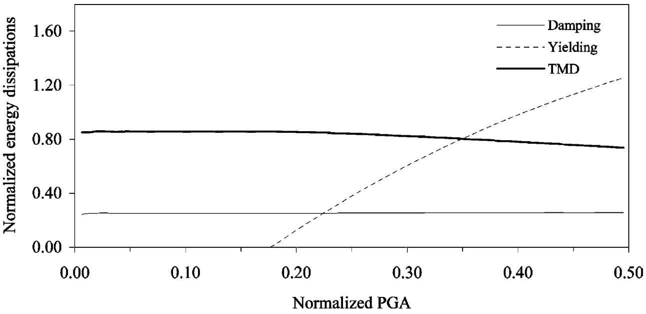

Fig. 8. Normalized energy dissipation in structure with TMD against normalized PGA (frequency ratio = 1.0).

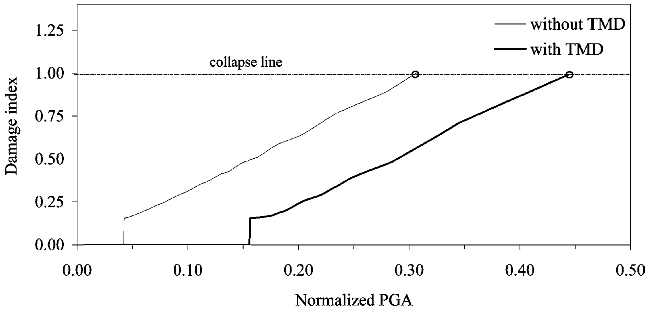

Fig. 9. Damage index of structure against normalized PGA (frequency ratio = 1.0).

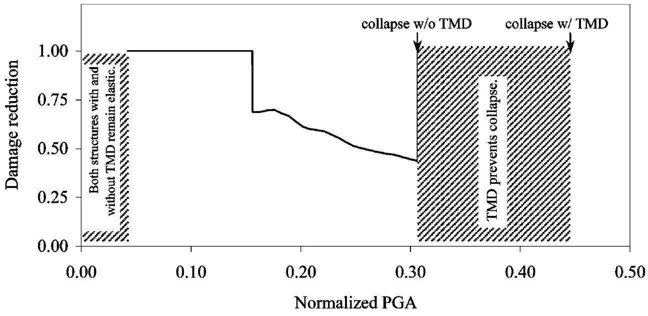

Fig. 10. Damage reduction of structure with TMD against normalized PGA (frequency ratio = 1.0).

malized PGA less than 0.042, the damage reduction of the structure due to TMD is not defined since there is no damage to the corresponding structure without TMD. Unlike the displacement reduction as in Fig. 7, the figure shows complete damage reduction due to TMD for normalized PGA between 0.042 and 0.156. This implies the usefulness of TMD even when the structure is subjected to moderate input ground motion. However, further increasing the PGA causes yielding in the structure with TMD and damage reduction of about 40–70% is obtained for normalized PGA between 0.156 and 0.306. In this range of input PGA, the damage reduction decreases as the PGA increases. It is also observed that, for normalized PGA between 0.306 and 0.443, the computed damage of the structure without TMD exceeds the collapse limit, i.e. $DI > 1.0$, while that of the structure with TMD is still within the limit. This corresponds to the case where the TMD can prevent the structure from collapse. Beyond this range of input PGA (normalized PGA greater than 0.443), the structure with TMD also collapses and therefore the application of TMD to the structure provides no benefit.

8. Effectiveness of TMD under a distant earthquake

In this section, ground motion recorded from the distant earthquake in Mexico City (1985) at SCT station in the N-S direction is employed as the input ground excitation. This record is found to have a predominant period close to the first natural period of the structure. The effectiveness of TMD is again evaluated in terms of displacement and damage reduction of the structure.

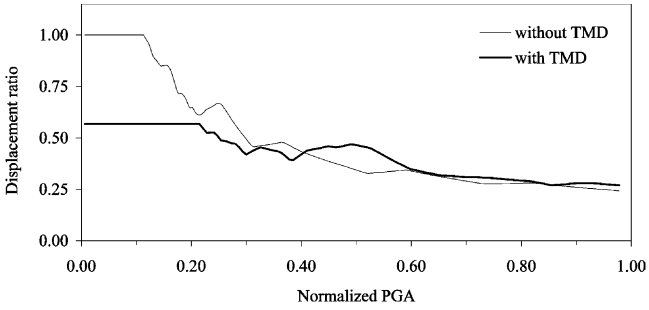

Fig. 11. Displacement ratio of the structure against normalized PGA (under far-field earthquake excitation).

8.1. Displacement reduction

Fig. 11 plots the displacement ratio of the structure with and without TMD against the input PGA of ground motion. In this figure, the displacement ratios are calculated using peak displacement instead of steady-state displacement as in Fig. 6. It can be seen from the figure that TMD is very effective in reducing the peak displacement of the structure when the structure is vibrated within the elastic range. However, its effectiveness gradually decreases as the inelasticity in the structure increases. It is noticed that, for normalized PGA higher than 0.40, the structure with TMD exhibits slightly larger displacement than that without TMD. This leads to negative displacement reduction of TMD as shown in Fig. 12.

8.2. Damage reduction

Fig. 13 compares the damage indices of the structure with and without TMD under the SCT ground motion for various PGAs, while Fig. 14 shows the damage reduction gained from the application of TMD. The figures reveal that TMD always reduces the damage to the

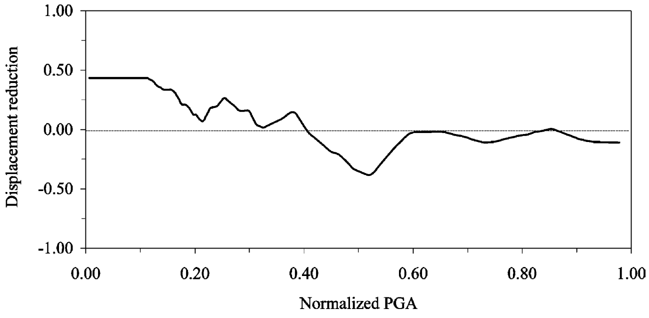

Fig. 12. Displacement reduction of the structure with TMD against normalized PGA(under far-field earthquake excitation).

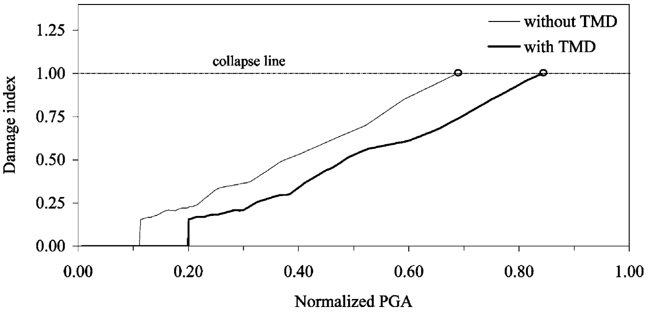

Fig. 13. Damage index of structure against normalized PGA (under far-field earthquake excitation).

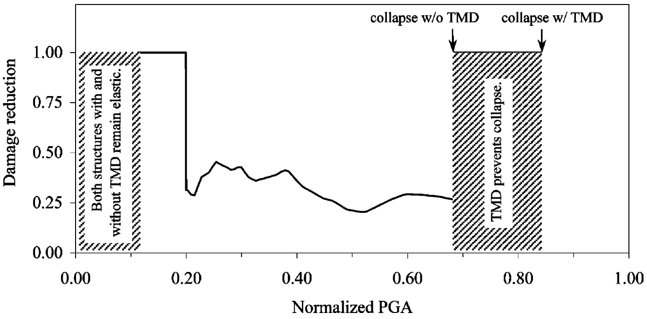

Fig. 14. Damage reduction of structure with TMD against normalized PGA (under far-field earthquake excitation).

structure for the entire range of PGA considered. Similar to the case of the harmonic ground motion, TMD perfectly protects the structure from damage for normalized PGA less than 0.20, and yields about 20–40% reduction of the damage for normalized PGA from 0.20 to 0.685. It is interesting to note that, for normalized PGA greater than 0.40, TMD provides positive effectiveness in damage reduction of the structure although its effectiveness in terms of displacement reduction is found to be negative (see also Fig. 12). This is because the damage index of the structure under the SCT ground motion is mainly governed by the energy term, i.e. the second term of Eq. (4), which is effectively reduced by TMD. It should also be noted that, for this ground excitation, TMD prevents collapse for normalized PGA from 0.685 to 0.841. Beyond this range of the input PGA, the structure with TMD collapses and therefore the application of TMD becomes useless.

9. Conclusions

The effectiveness of the TMD for control of the 20-storey reinforced concrete building subjected to both harmonic and the 1985 Mexico City (SCT) ground motions is investigated. Using numerical simulation, the obtained results indicate that the commonly used displacement reduction of the structure fails to prescribe the effectiveness of the TMD when damage occurs in the structure. This is because it does not provide sufficient information on the damage state of the controlled structure which is the main concern in engineering applications.

Therefore, direct use of damage reduction of the structure becomes more relevant and it is employed as the indicator of TMD effectiveness in this paper. Based on the ground motions considered, which have predominant periods close to that of the structure, the TMD is found to provide damage protection for the structure up to a certain level of input PGA. With a higher level of input PGA, it also significantly reduces the damage to the structure, which would suffer substantial damage in its absence. With further increase of input PGA, it is found that the application of TMD can prevent the structure from collapse. By comparing these damage characteristics to those of the structure with higher yield strength of base shear, this application of TMD is equivalent to an increase in the yield strength of the structure of about 45 and 20% for the harmonic and the SCT ground motions, respectively. This is of practical significance in view of the current trend toward performance based design in which the TMD may be installed to achieve the target performance of buildings designed for seismic loads.

Acknowledgements

The authors are grateful to the Thailand Research Fund (TRF) for the research scholar grant for this project. They would like to thank Mr. P. Vaijunya for providing the design of the example structure used in this study.

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